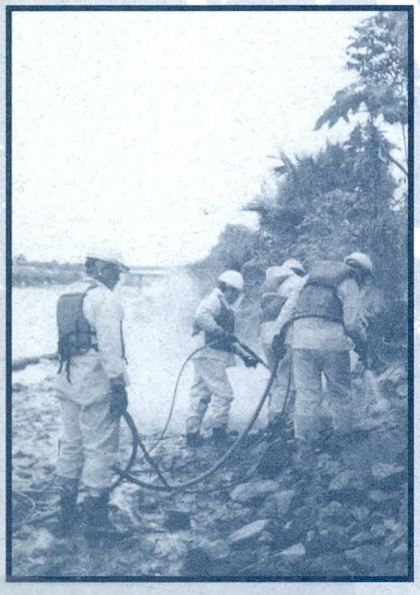
GEOTECHNICAL EVALUATION
BETHANY HOME OUTFALL
CHANNEL REACH D
PHOENIX, ARIZONA



Geotechnical
and
Environmental
Sciences
Consultants

Ninyo & Moore

GEOTECHNICAL EVALUATION
BETHANY HOME OUTFALL
CHANNEL REACH D
PHOENIX, ARIZONA

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March 26, 2008
(Revised September 16, 2008)
Project No. 601850001

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Subject: Geotechnical Evaluation
Bethany Home Outfall Channel Reach D
Phoenix, Arizona

Dear Mr. Ford:

In accordance with your request, Ninyo & Moore has revised our geotechnical evaluation for the above-referenced site. The attached revised report presents our methodology, findings, conclusions, and recommendations regarding the geotechnical conditions at the project site.

We appreciate the opportunity to be of service to you during this phase of the project. If you have any questions or comments regarding this report, please call at your convenience.

Respectfully submitted,
NINYO & MOORE

Registered Professional Engineer (Civil) CERTIFICATE NO. 41716 KEVIN L. PORTER Date Signed 9/16/08 ARIZONA, U.S.A.

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KLP/SAH/SDN/zvs

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TABLE OF CONTENTS

Page

1. INTRODUCTION 1
2. SCOPE OF SERVICES 1
3. SITE DESCRIPTION 2
4. PROPOSED CONSTRUCTION 2
5. FIELD EXPLORATION AND LABORATORY TESTING 3
6. GEOLOGY AND SUBSURFACE CONDITIONS 3
   6.1. Geologic Setting 4
   6.2. Subsurface Conditions 4
      6.2.1. Asphaltic Concrete and Aggregate Base 4
      6.2.2. Fill 5
      6.2.3. Alluvium 5
      6.2.4. Groundwater 5
7. GEOLOGIC HAZARDS 5
   7.1. Land Subsidence and Earth Fissures 6
   7.2. Faulting and Seismicity 6
   7.3. Liquefaction Potential 7
8. CONCLUSIONS 7
9. RECOMMENDATIONS 8
   9.1. Earthwork 9
      9.1.1. Excavations 9
      9.1.2. Grading, Fill Placement, and Compaction 9
      9.1.3. Imported Fill Material 11
   9.2. Temporary Cut Slopes 11
   9.3. Temporary Earth Retention Systems 12
   9.4. Below-Grade Walls 15
   9.5. Construction Dewatering 15
   9.6. Corrosion 16
   9.7. Concrete 17
   9.8. Pavements 18
   9.9. Site Drainage 19
   9.10. Instrumentation and Documentation 19
      9.10.1. Documentation of Existing Conditions 19
      9.10.2. Lateral Movement of Shoring Support System 20
      9.10.3. Ground Surface Settlement 20
   9.11. Pre-Construction Conference 20
   9.12. Construction Observation and Testing 20
10. LIMITATIONS 21

11. SELECTED REFERENCES 23

Tables

Table 1 – Seismic Design Parameters 7
Table 2 – UBC Requirements for Concrete Exposed to Sulfate-Containing Soil 17

Figures

Figure 1 – Site Location Map
Figure 2 – Boring Location Map
Figure 3 – Lateral Earth Pressures for Temporary Cantilevered Shoring
Figure 4 – Lateral Earth Pressures for Braced Excavation
Figure 5 – Lateral Earth Pressures for Underground Structures

Appendices

Appendix A – Boring Logs
Appendix B – Laboratory Testing

1. INTRODUCTION

In accordance with your authorization, we have performed a geotechnical evaluation for the proposed Bethany Home Outfall Channel Reach D project in Phoenix, Arizona. The purpose of our evaluation was to assess the subsurface conditions at the project site in order to formulate geotechnical recommendations for design and construction. This report presents the results of our evaluation and our geotechnical conclusions and recommendations regarding the proposed construction.

2. SCOPE OF SERVICES

The scope of our services for the project generally included:

- Reviewing readily available aerial photographs and published geologic literature, including maps and reports pertaining to the project site and vicinity.
- Marking out the boring locations, obtaining needed permits, and notifying Arizona Blue Stake of the boring locations prior to drilling.
- Coring the existing pavement at 67th Avenue and Indian School Road to evaluate the existing pavement sections.
- Drilling, logging, and sampling five small-diameter exploratory borings that extended approximately 20 to 40 feet below ground surface (bgs). The boring logs are presented in Appendix A.
- Performing laboratory tests of selected samples obtained from the borings to evaluate in-situ moisture content and dry density, sieve analyses, Atterberg limits, maximum dry density and optimum moisture content, and corrosivity characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides). The results of the laboratory testing are presented on the boring logs and in Appendix B.
- Preparing this report presenting our findings, conclusions, and recommendations regarding the design and construction of the project.

Our scope of services did not include environmental consulting services, such as hazardous waste sampling or analytical testing, at the site. A detailed scope of services and estimated fee for such services can be provided upon request.

3. SITE DESCRIPTION

The project site is located in Section 19 of Township 2 North, Range 2 East. The project is located within the City of Phoenix, Arizona, extending along the northern bank of the Grand Canal from west of 67th Avenue to south of Indian School Road. The approximate location of the site is depicted on the Site Location Map (Figure 1). At the time of our evaluation, the planned outfall channel alignment was generally bounded by private and residential properties along the northeast, paralleled the existing SRP Grand Canal, which is located along the southwesterly side, and crosses 67th Avenue and Indian School Road to the north and south, respectively.

According to the *Fowler, Arizona 7.5-Minute United States Geological Survey (USGS) Topographic Quadrangle Map (1982),* the site lies at an average elevation of roughly 1,100 feet relative to mean sea level (MSL). Based on the information from the quadrangle map, the project site generally slopes from the northeast to the southwest.

Six aerial photographs from the Flood Control District of Maricopa County were reviewed for this project. Aerial photographs from 1937, 1949, and 1959 depict the project site as a lined canal cross cutting agricultural land. Scattered residential structures were noted near the site. A 1964 aerial photograph showed the agricultural land north of the Grand Canal as being developed with residential structures and paved roadways. Aerial photographs from 1993 and 2007 depicted residential development along the north side of the canal, and a commercial structure and parking lot along the south side of the canal, similar to its current condition.

4. PROPOSED CONSTRUCTION

The project will generally consist of the design and construction of the Bethany Home Outfall Channel from 67th Avenue to Indian School Road in Phoenix, Arizona. The planned outfall channel is anticipated to be concrete pipe situated approximately 10 to 15 feet below existing grade and generally sloping from southeast to northwest. The outfall channel will connect the existing Sunset Retention Basin, located at the southeast end of the alignment, to the existing Maryvale Basin, located at the northwest end of the alignment.

We also understand there are existing utilities located within the proposed channel alignment which may need to be relocated.

5. FIELD EXPLORATION AND LABORATORY TESTING

On August 14 and 20, 2007, Ninyo & Moore conducted a subsurface exploration, which consisted of the drilling, logging, and sampling of five small-diameter borings, along the proposed storm drain alignment, as depicted on the Boring Location Map (Figure 2). The borings were drilled using a CME-75 truck-mounted drill rig equipped with hollow-stem augers. The borings, denoted as B-1 through B-5, extended approximately 20 to 40 feet bgs. Bulk and relatively undisturbed soil samples were collected at selected intervals. Detailed descriptions of the soils encountered at each boring location are presented on the boring logs in Appendix A.

Ninyo & Moore personnel logged the borings in general accordance with the Unified Soil Classification System (USCS) and American Society for Testing and Materials (ASTM) D 2488) by observing cuttings and drive samples. Collected ring samples were trimmed in the field, wrapped in plastic bags, and placed in cylindrical plastic containers to retain in-place moisture conditions. Similarly, the Standard Penetration Test and bulk samples were sealed in plastic bags to retain their approximate in-place moisture.

The soil samples collected from our field activities were transported to the Ninyo & Moore laboratory in Phoenix, Arizona for geotechnical laboratory analysis. The laboratory testing included evaluation of in-situ moisture content and dry density, grain-size distribution, Atterberg limits, maximum dry density and optimum moisture, and corrosion characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides). The results of the laboratory tests are presented on the boring logs in Appendix A and in Appendix B. Appendix B also describes in detail each laboratory tests performed.

6. GEOLOGY AND SUBSURFACE CONDITIONS

The geology and subsurface conditions at the site are described in the following sections.

6.1. Geologic Setting

The project site is located in the Sonoran Desert Section of the Basin and Range physiographic province, which is typified by broad alluvial valleys separated by steep, discontinuous, subparallel mountain ranges. The mountain ranges generally trend north-south and northwest-southeast. The basin floors consist of alluvium with thickness extending to several thousands of feet.

The basins and surrounding mountains were formed approximately 10 to 18 million years ago during the mid- to late-Tertiary. Extensional tectonics resulted in the formation of horsts (mountains) and grabens (basins) with vertical displacement along high-angle normal faults. Intermittent volcanic activity also occurred during this time. The surrounding basins filled with alluvium from the erosion of the surrounding mountains as well as from deposition from rivers. Coarser-grained alluvial material was deposited at the margins of the basins near the mountains.

The surficial geology of the site is generally described as being Holocene (<10,000 years old) alluvial terrace and fan deposits, generally consisting of well-sorted (poorly graded) sand with silt. Very little soil development has occurred in this deposit. Stage I to Stage II caliche cementation is common within this unit (Demsey, 1988).

6.2. Subsurface Conditions

Our knowledge of the subsurface conditions at the project site is based on our field exploration and laboratory testing, and our understanding of the general geology of the area. The following sections provide a generalized description of the materials encountered. More detailed descriptions are presented on the boring logs in Appendix A.

6.2.1. Asphaltic Concrete and Aggregate Base

Asphaltic concrete (AC), which ranged in thickness from about 2 to 11 inches, was encountered at the surface in borings B-1, B-4 and B-5. Aggregate base (AB) was encountered below the AC in these borings and typically consisted of silty gravel with sand and ranged in thickness from about 3.5 to 8 inches.

6.2.2. Fill

Fill soils were encountered at the ground surface in boring B-3 and below the AB material in borings B-1 and B-5. The fill generally consisted of clayey sand, sandy clay, poorly graded sand, and sandy gravel in our borings.

6.2.3. Alluvium

Alluvium was encountered below the fill or AB and extended to the total depths explored. The alluvium generally consisted of silty sand, sandy silt, clayey sand, sand, sandy clay, and sandy gravel in our borings.

6.2.4. Groundwater

Groundwater was not encountered in our exploratory borings. Based on well data from the Arizona Department of Water Resources (ADWR), the approximate depth to regional groundwater has been estimated to be as shallow as 140 feet bgs. Groundwater levels can fluctuate due to seasonal variations, potential seepage from the adjacent canal, irrigation, groundwater withdrawal or injection, and other factors. In general, groundwater is not expected to be a constraint to the construction of the project; however, any cracks or flaws in the concrete lining of the adjacent canal could result in seepage into any excavations below the canal water level.

7. GEOLOGIC HAZARDS

The following sections describe regional geologic hazards, including land subsidence, earth fissures, and seismicity.

7.1. Land Subsidence and Earth Fissures

Groundwater depletion, due to groundwater pumping, has caused land subsidence and earth fissures in numerous alluvial basins in Arizona. It has been estimated that subsidence has affected more than 3,000 square miles and has caused damage to a variety of engineered structures and agricultural land (Schumann and Genualdi, 1986). Since 1948, excessive groundwater withdrawal has been documented in several alluvial valleys where groundwater levels have been reportedly lowered by up to 500 feet. With such large depletions of groundwater, the alluvium has undergone consolidation, resulting in large areas of land subsidence.

In Arizona, earth fissures are generally associated with land subsidence and pose an ongoing geologic hazard. Earth fissures generally form near the margins of geomorphic basins where significant amounts of groundwater depletion have occurred. Reportedly, earth fissures have also formed due to tensional stress caused by differential subsidence of the unconsolidated alluvial materials over buried bedrock ridges and irregular bedrock surfaces (Schumann and Genualdi, 1986).

Based on our field reconnaissance and review of the referenced material, there are no known earth-fissures underlying the subject site. Based on our research, the closest documented earth fissure to this site is approximately 13 miles to the northwest. Therefore, land subsidence and earth fissures are not expected to be a constraint to this project. However, continued groundwater withdrawal in the area may result in subsidence of the valley and the formation of new fissures, or the extension of existing fissures, and the prediction of the location of new fissures or subsidence bowls can not accurately be predicted.

7.2. Faulting and Seismicity

The site lies within the Sonoran zone, which is a relatively stable tectonic region located in southwestern Arizona, southeastern California, southern Nevada, and northern Mexico (Euge et al., 1992). This zone is characterized by sparse seismicity and few Quaternary faults. Based on our field observations and on our review of readily available published geological maps and literature, there are no known active faults underlying the subject site or adjacent areas. The closest Quaternary fault to the site is the Carefree fault zone, located approximately 30 miles to the northeast of the site (Pearthree, 1998). Approximately 2 meters of displacement has occurred along this fault within middle Pleistocene deposits (<750,000 years), but the upper Pleistocene and Holocene deposits (<250,000 years) are not displaced.

Based on a probabilistic seismic hazard assessment for the Western United States, issued by the USGS (2000), the site is located in a zone where the peak ground accelerations that have a 10 percent, and 2 percent probability of being exceeded in 50 years are 0.04g and 0.08g, respectively. These ground motion values are calculated for "firm rock" sites, which correspond to a shear-wave velocity of approximately 2,500 feet per second in approximately the top 100 feet bgs. Different soil or rock types may amplify or de-amplify these values. Seismic design parameters according to the 2003 International Building Code (IBC) are presented in Table 1.

Table 1 – Seismic Design Parameters

Parameter	Value	2003 IBC Reference
Site Coefficient, F_a	1.6	Table 1615.1.2(1)
Site Coefficient, F_v	2.4	Table 1615.1.2(2)
Site Class Definition	D	Table 1615.1.1

7.3. Liquefaction Potential

Considering the density and consistency of the soils encountered at the site during our subsurface evaluation, the lack of near-surface water, and the low ground motion hazard (relatively low peak ground accelerations), the likelihood or potential for liquefaction is considered to be negligible, and, therefore, liquefaction is not a design consideration.

8. CONCLUSIONS

Based on the results of our subsurface evaluation, laboratory testing, and data analysis, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations presented in this report are incorporated into the design and construction of the proposed project, as appropriate. Geotechnical considerations include the following:

- The on-site soils should generally be excavatable to the proposed excavation depths, with earth moving construction equipment in good working condition. However, some locations of very dense materials were encountered in our borings, which could slow the rate of excavation.

- Due to interbedded layers of sandy material, possible vibrations near open excavations (due to the adjacent roadway and construction activity), and the potential consequence of slope instability, an Occupational Safety and Health Administration (OSHA) soil-type "C" should be used for planning excavation side slopes.

- Because the proposed below-grade structure is to be in close proximity to other existing structures, a temporary earth retention system may be needed to maintain the stability and integrity of the sides of the excavation during construction of the planned improvements.

- We estimate an earthwork (shrinkage) factor of 15 to 20 percent for this project.

- New grade slabs, pavements and flatwork should also be founded in engineered fill soil.

- Imported soils and soils generated from on-site excavation activities that exhibit low plasticity and very low to low expansion potential can generally be used as engineered fill.

- Corrosivity test results indicate that subgrade soils at the site may be corrosive to ferrous metals, and the sulfate content of the soils present a negligible sulfate exposure to concrete.

- Groundwater was not observed in our exploratory borings. The static groundwater table at the site is anticipated to be approximately 140 feet bgs or deeper. However, seepage from the adjacent canal, if present, could cause perched groundwater conditions and/or wet soils and should be anticipated.

- No known or documented geologic hazards are present underlying or immediately adjacent to the site.

9. RECOMMENDATIONS

The following sections present our geotechnical recommendations for the proposed construction. In general, the recommendations and guidelines outlined in the Maricopa Association of Governments (MAG) Standard Specifications and Details (2005) should be used except where different in this report. If the proposed construction is changed from that discussed in this report, Ninyo & Moore should be contacted for additional recommendations.

9.1. Earthwork

The following sections provide our earthwork recommendations.

9.1.1. Excavations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our exploratory borings, our site observations, and our experience with similar materials. In our opinion, excavation of the on-site materials, to the anticipated depths, can generally be accomplished with excavation equipment in good operating condition. However, some locations of very dense materials were encountered in our borings, which could slow the rate of excavation.

The contractor should provide a safely sloped excavation or an adequately constructed and braced shoring system, in compliance with OSHA regulations, for employees working in an excavation that may expose them to the danger of moving ground. If construction or earth material is stored or equipment is operated near an excavation, flatter slope geometry or stronger shoring should be used during construction. Discussions related to temporary sloped excavations and earth retention systems are presented in Sections 9.2 and 9.3 of this report, respectively.

9.1.2. Grading, Fill Placement, and Compaction

Demolition debris should be removed from the site and disposed of at a legal dumpsite. In pavement and exterior flatwork areas, obstructions that extend below finish grade, if present, should be removed and the resulting holes filled with compacted soil. Also in these areas, the geotechnical consultant should carefully evaluate any areas of loose or soft and wet soils prior to placement of grade-raise fill or other construction. Drying or overexcavation of some materials may be appropriate, in addition to the earthwork preparation recommendations presented below.

Soils generated from on-site excavation activities that exhibit relatively low plasticity indices and very low to low expansion potential are generally suitable for reuse as engineered fill. Relatively low plasticity indices are defined as a Plasticity Index (by ASTM

4318) value of 20 or less. Very low to low expansion potential soils are defined as having an Expansion Index (by ASTM D 4829) of 50 or less. Our two Atterberg Limits tests indicated plasticity indices ranging from 22 to 26. As such, some of the on-site soils may not be suitable for re-use as engineered fill during construction.

In addition, fill soils should not include organic material (greater than about 4 percent organic content), clay lumps, construction debris, rock particles, and other non-soil fill materials larger than 6 inches in dimension. This material should be disposed of offsite or in non-structural areas. An earthwork (shrinkage) factor of 15 to 20 percent for the on-site soils is estimated.

Due to the presence of relatively loose and/or soft supporting soils, the new outfall structure should be founded on a zone of adequately moisture-conditioned and re-compacted engineered fill. This improved zone may be accomplished by overexcavating to a depth of 1 foot and scarifying an additional 1 foot beneath the bearing elevation. However, in areas where this creates a conflict with existing utilities, the improved zone should extend to the top of the existing utility. The improved zone associated with this structure should also extend horizontally by 2 feet or more beyond the foundation footprint. The engineered fill should be placed in horizontal lifts no more than 9 inches in loose thickness and compacted by appropriate mechanical methods to 95 percent, or more, relative compaction in accordance with ASTM D698 and at a moisture content slightly above its laboratory optimum.

In addition, we recommend that the new grade slabs, pavements and exterior flatwork that are situated near the existing ground surface be supported on 1 foot or more of adequately moisture-conditioned and compacted structural fill. The fill thickness should be measured from the bottom of the AB layer and should be compacted by appropriate mechanical methods to 95 percent, or more, relative compaction in accordance with ASTM D698 and at a moisture content slightly above its laboratory optimum. Acceptance criteria for imported fill materials were provided above.

Following the overexcavation as described above, and prior to the placement of new fill, the resulting exposed surface should be carefully evaluated by the geotechnical consultant. This evaluation could consist of proof-rolling, soil probing, visual assessment and/or additional laboratory testing. Based on this evaluation, additional remediation may be needed, which could include additional improvement of the exposed surface. This additional remediation (if needed) should be resolved by the geotechnical consultant during the earthwork operations.

Backfill material used in excavations should be moisture conditioned to a moisture slightly above its laboratory optimum moisture content. Backfill should be mechanically compacted to a relative compaction of 95 percent or more as evaluated by ASTM D698. Lift thickness for backfill will be dependent on the type of compaction equipment utilized, but should generally be placed in uniform lifts not exceeding 9 inches in loose thickness. Special care should be exercised to avoid damaging the pipe or other structures during the compaction of the backfill.

9.1.3. Imported Fill Material

Imported fill, if utilized, should consist of granular material with low plasticity and a very low to low expansion potential. Import material in contact with ferrous metals should have low corrosion potential (minimum resistivity more than 2,000 ohm-cm, chloride content less than 25 parts per million (ppm)). Import material in contact with concrete should have a soluble sulfate content of less than 0.1 percent. The geotechnical consultant should evaluate such materials and details of their placement prior to importation.

9.2. Temporary Cut Slopes

Based on the information we received from your office, we understand that temporary excavations will be needed for this project to construct the below-grade structure. Based on the subsurface information obtained from our exploratory borings and our experience with similar projects, we recommend that cut slopes associated with temporary excavations be constructed at a slope ratio no steeper than 1.5:1 (horizontal to vertical) up to a height of 20 feet. If the depth of excavation exceeds 20 feet, the cut slopes should be re-evaluated. These cut slope recommendations assume that no groundwater is present and no surcharge loading will be located adjacent to the top of the cut. If existing structures or utilities are located near the proposed excavation, such that a sloped excavation would not be appropriate, we recommend a temporary retention system be incorporated as an alternative to a sloped excavation. Recommendations related to temporary retention systems are presented below.

9.3. Temporary Earth Retention Systems

Based on our understanding of the proposed construction, temporary excavations up to an average depth of approximately 10 feet below the existing grade may be needed to accommodate the below-grade structure. Due to the proximity of the proposed construction to existing structures (e.g., Grand canal, residences, various utilities), it may not be feasible to perform the excavation at a stable slope ratio without causing loss of lateral support to the adjacent improvements. For this reason, temporary shoring may be needed for some portions of the proposed construction of the below-grade structure. The temporary shoring system may be incorporated into a permanent retaining structure upon excavating down to the subgrade elevation.

The shoring system should be designed for a minimum safety factor of 1.5, and the lateral deformation of the ground surface should be managed by structural design in order to protect the adjacent structures. The shoring should be designed to support the surcharge loads from the adjacent structures in addition to the earth pressures exerted by the native backfill soils. Recommended design values with respect to distribution of earth pressures on shoring elements are presented below.

Shoring systems such as soldier piles and lagging, sheet piles, or soil nails may be used to support the sidewalls of the temporary excavations as needed along the perimeter of the proposed construction. In areas where sensitive structures are located close to the proposed excavations, driven piles may not be a viable alternative due to the possible damage that could occur to nearby structures while the piles are being driven or vibrated into place. In comparison, soldier piles and lagging (wood or precast concrete) or soil nails are considered to offer a more practical and safe method of shoring within the site.

The soldier piles should be on the order of 24 inches in diameter. The final depth and spacing of the piles should be evaluated by the project structural engineer based on the estimated total service (dead and live) and lateral loads. However, the piles should extend on the order of 15 feet below the bottom of the excavation. Once the soldier piles are installed and the concrete is cured, excavation of the site may begin. Care should be taken to ensure that the lagging drops down as the excavation advances to lower elevations. Any gaps in the lagging could cause undermining of the adjacent structures.

The excavations for the soldier piles should be observed by a representative of the project geotechnical consultant to verify total embedment depths determined by the project structural engineer. The drilled holes should be cleared of loose soil and/or construction debris prior to pouring concrete. The excavation should be conducted with continuous monitoring of the retained soil and all the adjacent structures for any signs of potential lateral and vertical movements. If any movement is observed, it should be brought to the immediate attention of the project geotechnical engineer and the excavation suspended until appropriate corrective measures are taken.

For design of cantilever temporary shoring, the lateral earth pressures noted on Figure 3 should be used. For cantilever soldier piles, an active earth pressure of 35 pounds per square foot (psf) per foot of depth may be considered for the backfill materials. It should be noted that under this condition, movement of the soldier piles are not restrained so that the soil internal strength can be fully developed. If the soldier piles or shoring system is braced using struts or tie-backs, the lateral earth pressures in Figure 4 should be used. A passive earth pressure increasing at a rate of 300 psf per foot of depth, to a value of 3,000 or more psf per foot of depth, may be used to estimate lateral resistance for soldier piles. The passive resistance should be ignored for the upper one diameter of the soldier pile embedded below the subgrade level.

Soil nail retaining walls are common in the Phoenix area. Soil nailing involves reinforcing and strengthening the existing ground by installing closely-spaced steel bars, called "nails," into an excavation face as construction proceeds from the "top down." Soil nailed structures behave like gravity walls and are typically installed in 5 foot excavation lifts.

For a conventional soil nail wall face with horizontal ground behind the wall, the nail lengths would be estimated to be 0.5 to 0.8 times the height of the wall. In some cases, the nail lengths could be 70 to 100 percent of the wall height. The soil nail spacing could range from about 3 to 6 feet, but is typically in the 5 to 6 foot range. The bars are usually installed at a 10 to 20 degree angle below the horizontal plane. After installation, the space between the bar and drill hole is grouted with cement grout. Reinforcement consisting of wire mesh or walers composed of reinforcing bars, is then installed across the excavation face between the nails. Typically a shotcrete face is then applied.

If seepage behind the soil nail wall is anticipated, a drainage system is provided along the soil excavation face. This drainage system could consist of prefabricated drains. The drain mats should be extended down the full height of the excavation, and discharge into a collector pipe at the bottom of the excavation, suitably outletted.

One advantage of soil nailed walls is their relative ease of construction and reduced construction time. Construction equipment is relatively small, mobile and quiet when compared to soldier pile installations. Soil nail wall movements are typically higher than movements from soldier piles and lagging. If soil nails are used, knowledge of easement rights and avoidance of existing utilities and structures is imperative. Due to the lateral extent of the soil nails into the adjacent ground, we do not recommend the use of soils nails if right-of-way or limited lateral space is a constraint.

The contractor performing the earth retention system work should have 5 or more years of experience in temporary earth retention systems and provide evidence of similar projects. Geotechnical information, including parameters for design, may be developed from the subsurface information provided herein. Due to the sensitive nature of adjacent structures, we also recommend a monitoring program be developed. This monitoring program may be performed by the contractor or an independent representative of the Owner.

9.4. Below-Grade Walls

For below-grade structures that are rigidly restrained so as not to rotate sufficiently to reach active earth pressure conditions, at-rest pressure conditions will exist. We anticipate below-grade walls will be in an at-rest state. For at-rest earth pressure conditions, an equivalent fluid unit weight of 55 pcf should be used for the drained condition as depicted in Figure 5.

For passive resistance to lateral loads, we recommend that an equivalent fluid weight of 300 pcf be used up to a value of 3,000 psf. This value assumes that the ground is horizontal for a distance of 10 feet or more behind the wall or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 12 inches of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend that a coefficient of friction of 0.40 be used between soil and concrete. If passive and frictional resistances are to be used in combination, we recommend that the friction coefficient be reduced by two-thirds. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

Precautions should be considered to avoid overstressing below-grade walls during backfilling. Temporary bracing of the walls during backfilling may be needed to help avoid this problem.

9.5. Construction Dewatering

A shallow groundwater table is not anticipated along the alignment during construction. However, as indicated above, seepage and surface run-off could be anticipated near the Grand Canal or other drainage features. The need for dewatering should be considered along the alignment near these features. Surface run-off will vary seasonally depending on local rainfall.

Given the low possibility of encountering significant seepage along the alignment, we anticipate that the excavations that do encounter nuisance seepage or surface run-off could be dewatered by sumping the water from the bottom of the excavation. However, saturated sands, if encountered, may need more aggressive means of dewatering such as well points. If discharged water from the excavations is diverted to natural drainage channels, a special permit may be needed.

9.6. Corrosion

The corrosion potential of the on-site soil materials was analyzed to evaluate its potential effect on the foundations and structures. Corrosion potential was evaluated using the results of laboratory testing of a representative near-surface sample obtained during our subsurface evaluation.

Laboratory testing consisted of pH, minimum electrical resistivity, and soluble sulfate and chloride contents. The pH and minimum electrical resistivity tests were performed in general accordance with Arizona Test 236b, while sulfate and chloride tests were performed in accordance with Arizona Test 733 and 736, respectively. The results of the corrosivity tests are presented in Appendix B.

The soil pH value of the samples tested ranged from 7.8 to 8.5, which is considered to represent an alkaline environment. The minimum electrical resistivity measured in the laboratory ranged from 752 ohm-cm to 2,395 ohm-cm, which is considered corrosive to ferrous metals. The chloride content of the samples tested ranged from 43 ppm to 146 ppm, which is considered to have a potential corrosive effect to ferrous metals. The soluble sulfate content of the soil samples tested ranged from 0.001 percent to 0.006, which is considered to represent a negligible corrosive effect on exposed concrete.

The results of the laboratory testing indicate that the on-site soils may be corrosive to ferrous metals. Therefore, special consideration should be given to the use of heavy gauge, corrosion protected, underground steel pipe, if any are planned. As an alternative, wrapped/plastic pipe or reinforced concrete pipe should be considered. In addition, metallic elements used for excavation support systems will need to be protected from potentially corrosive soils. A corrosion specialist should be consulted for further recommendations.

9.7. Concrete

Laboratory chemical tests performed on an on-site soil samples indicated a sulfate content ranging from 0.001 percent to 0.006 percent by weight. Based on the following UBC table, the on-site soils should be considered to have a negligible sulfate exposure to concrete.

Table 2 – UBC Requirements for Concrete Exposed to Sulfate-Containing Soil

Sulfate Exposure	Water-Soluble Sulfate (SO_4) in Soil, Percentage by Weight	Cement Type	Water-Cementitious Materials Ratio, by Weight, Normal-Weight Aggregate Concrete[1]	f'_c, Normal-Weight and Lightweight Aggregate Concrete, psi x 0.00689 for MPa
Negligible	0.00 - 0.10	--	--	--
Moderate[2]	0.10 - 0.20	II, IP(MS), IS (MS)	0.50 or less	4,000 or more
Severe	0.20 - 2.00	V	0.45 or less	4,500 or more
Very severe	Over 2.00	V plus pozzolan[3]	0.45 or less	4,500 or more

[1] A lower water-cementitious materials ratio or higher strength may be needed for low permeability or for protection against corrosion of embedded items or freezing and thawing (Table 19-A-2).
[2] Seawater.
[3] Pozzolan that has been evaluated by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

Notwithstanding the sulfate test results and due to the limited number of chemical tests performed, as well as our experience with similar soil conditions and local practice, we recommend the use of Type II cement for construction of concrete structures at this site.

The concrete should have a water-cementitious materials ratio no more than 0.45 by weight for normal weight aggregate concrete. The structural engineer should ultimately select the concrete design strength based on the project specific loading conditions. However, higher strength concrete may be selected for increased durability, resistance to slab curling and shrinkage cracking.

9.8. Pavements

For the paved areas, we understand that AC pavement is considered for this project. The pavement sections recommended below are assumed to bear on imported or on-site soils with an average soil R-value of 30 or more.

The AASHTO method was also used to evaluate the asphalt pavement thicknesses recommended below. Specifically, the recommendations were based on the following input parameters:

Design Period:	20 years
Average Daily Traffic (2007)	38,000
Percent Trucks	5%
Percent Growth	5%
ESALs:	9,750,000
Reliability:	95 percent
Overall Deviation:	0.45
Soil Resilient Modulus:	18,000 psi
Initial Serviceability:	4.5
Terminal Serviceability:	2.5

An asphalt pavement section consisting of 6 or more inches of plant-mix asphalt (per MAG Section 710) over 10 or more inches of graded AB can be considered for pavement replacement along Indian School or 67th Avenue. For light duty areas (parking lots, driveways, etc), an asphalt pavement section consisting of 3 or more inches of plant-mix asphalt (per MAG Section 710) over 6 or more inches of graded AB can be utilized.

For asphalt pavement sections, we recommend the underlying subgrade soils be prepared as described in Section 9.1.2 of this report. AB material should be compacted to a relative compaction of 95 percent or more, as evaluated by ASTM D698, at a moisture content slightly above its laboratory optimum.

9.9. Site Drainage

Surface drainage should be provided to divert water away from buildings and off of paved surfaces. Surface water should not be permitted to drain toward the structures or to pond adjacent to footings or on pavement areas. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet or more away from the structures.

9.10. Instrumentation and Documentation

Consideration should be given to implementing documentation and instrumentation programs to evaluate design assumptions, existing conditions, and to monitor movements, levels, and deformations during construction. The monitoring programs may include the use of inclinometers, convergence points, and an array of surface control points. The resulting data should be reviewed and evaluated during construction. These programs should be in-place or conducted prior to the start of construction.

9.10.1. Documentation of Existing Conditions

We recommend that a pre-construction survey be performed prior to construction on residences and structures within approximately 50 feet of the proposed trench excavations. The pre-construction survey should consist of photographic documentation of the exterior portions of the buildings, including distress features, such as cracks and/or separations that may be present. Consideration may be given to videotaping the survey. In addition, interviews with owners should be conducted to provide knowledge of the age and type of the buildings as well as maintenance history and utility problems.

9.10.2. Lateral Movement of Shoring Support System

We recommend that inclinometers or survey points be established behind excavations located in areas where structures are located above a 1:1 (horizontal to vertical) plane projected from the bottom of the proposed excavations. The inclinometers or survey points should be monitored and evaluated daily during excavation activities to provide an advanced warning system of potential problems.

9.10.3. Ground Surface Settlement

We also recommend that an array of ground survey points be installed along the project alignment to monitor settlement. The survey points should be installed as close as practical to the excavation and incrementally away from the excavation. The contractor should be responsible for maintaining the total settlement beneath adjacent buildings to less than ½-inch. If settlements reach ¼-inch, we recommend that a review of the contractor's methods be performed and appropriate changes be made, if needed.

Consideration should be given to placing survey monitoring points on nearby structures to monitor the performance of the structures. In this way, a record of the performance of the structure will be maintained and available. This information, in conjunction with pre-construction surveys, is helpful in reducing potential claims and expediting and limiting settlement of legitimate claims.

9.11. Pre-Construction Conference

We recommend that a pre-construction conference be held. Representatives of the owner, civil engineer, the geotechnical consultant, and the contractor should be in attendance to discuss the project plans and schedule. Our office should be notified if the project description included herein is incorrect, or if the project characteristics are significantly changed.

9.12. Construction Observation and Testing

During construction operations, we recommend that a qualified geotechnical consultant perform observation and testing services for the project. These services should be performed to evaluate exposed subgrade conditions, including the extent and depth of overexcavation, to evaluate the suitability of proposed borrow materials for use as fill and to observe placement and test compaction of fill soils. If another geotechnical consultant is selected to perform observation and testing services for the project, we request that the selected consultant provide a letter to the owner, with a copy to Ninyo & Moore, indicating that they fully understand our recommendations and that they are in full agreement with the recommendations contained in this report. Qualified subcontractors utilizing appropriate techniques and construction materials should perform construction of the proposed improvements.

10. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

11. REFERENCES

American Concrete Institute, 2005, Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05).

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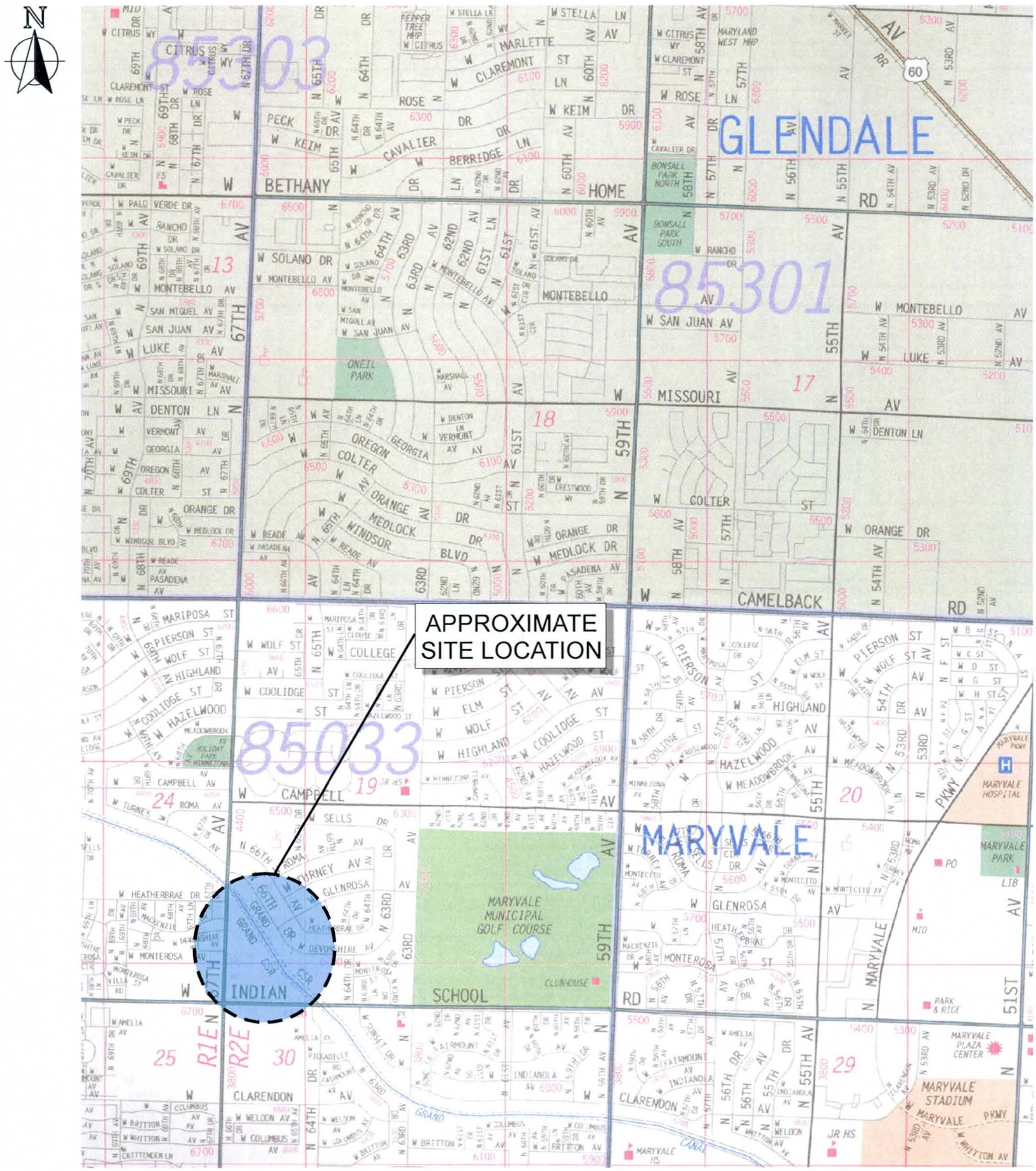


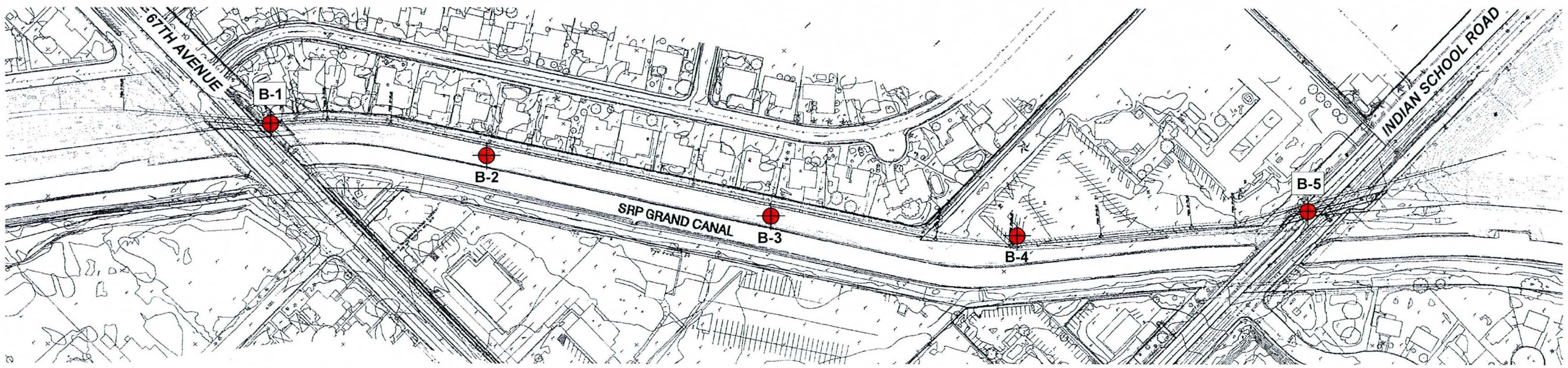
0 1900

Approximate Scale:
1 inch = 1900 feet

Source: Thomas Guide, Phoenix Metro Edition, 2006.

Ninyo & Moore		SITE LOCATION MAP	FIGURE 1
PROJECT NO: 601850001	DATE: 9/08	BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	

file no: 1850vmap1007



LEGEND

B-5 Approximate Boring Location

0 170

Approximate Scale:
1 inch = 170 feet

Source: Basemap modified after OLSSON ASSOCIATES, 2007.

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BORING LOCATION MAP

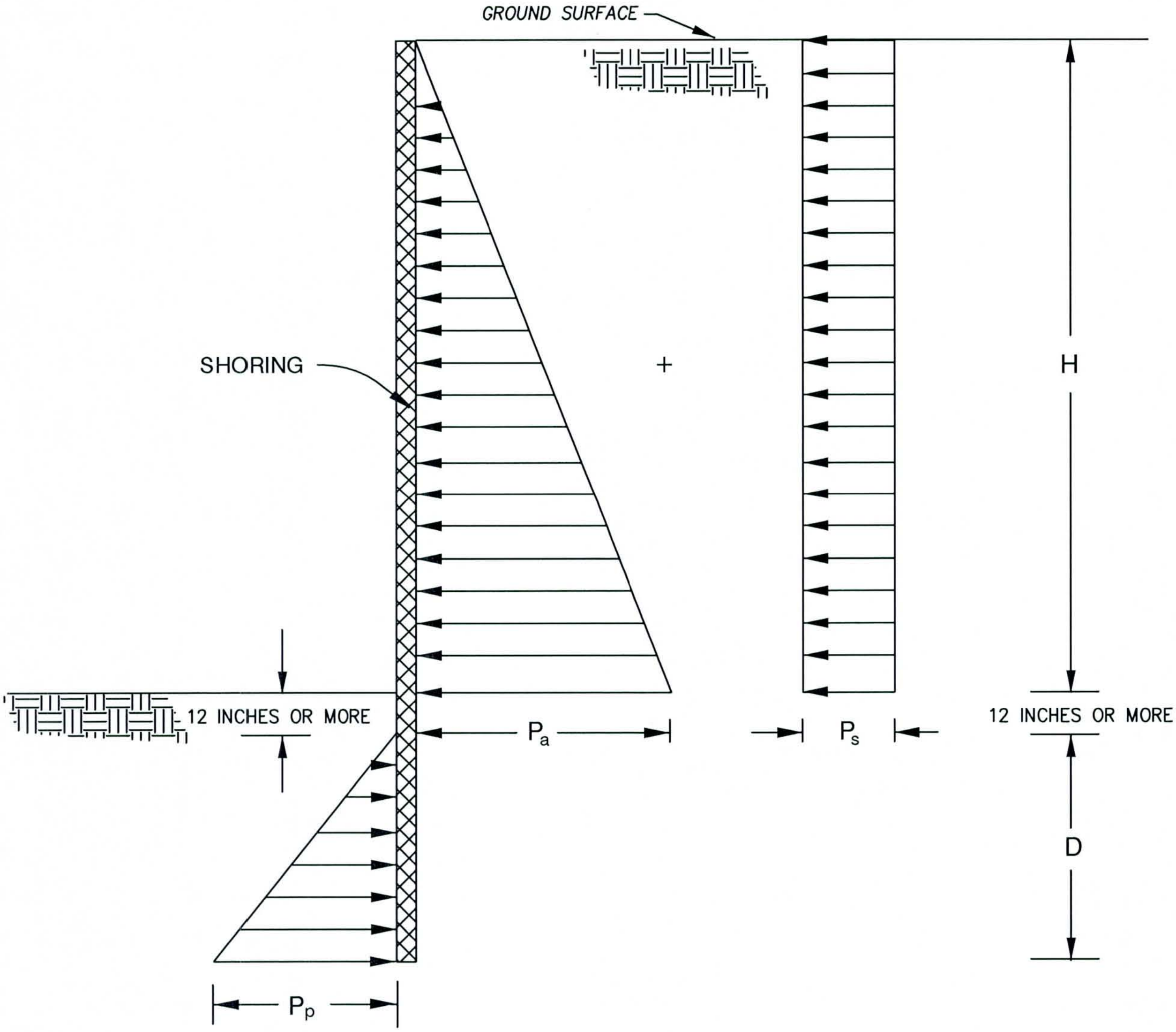
PROJECT NO: 601850001

DATE: 9/08

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

FIGURE 2

1850blm1007



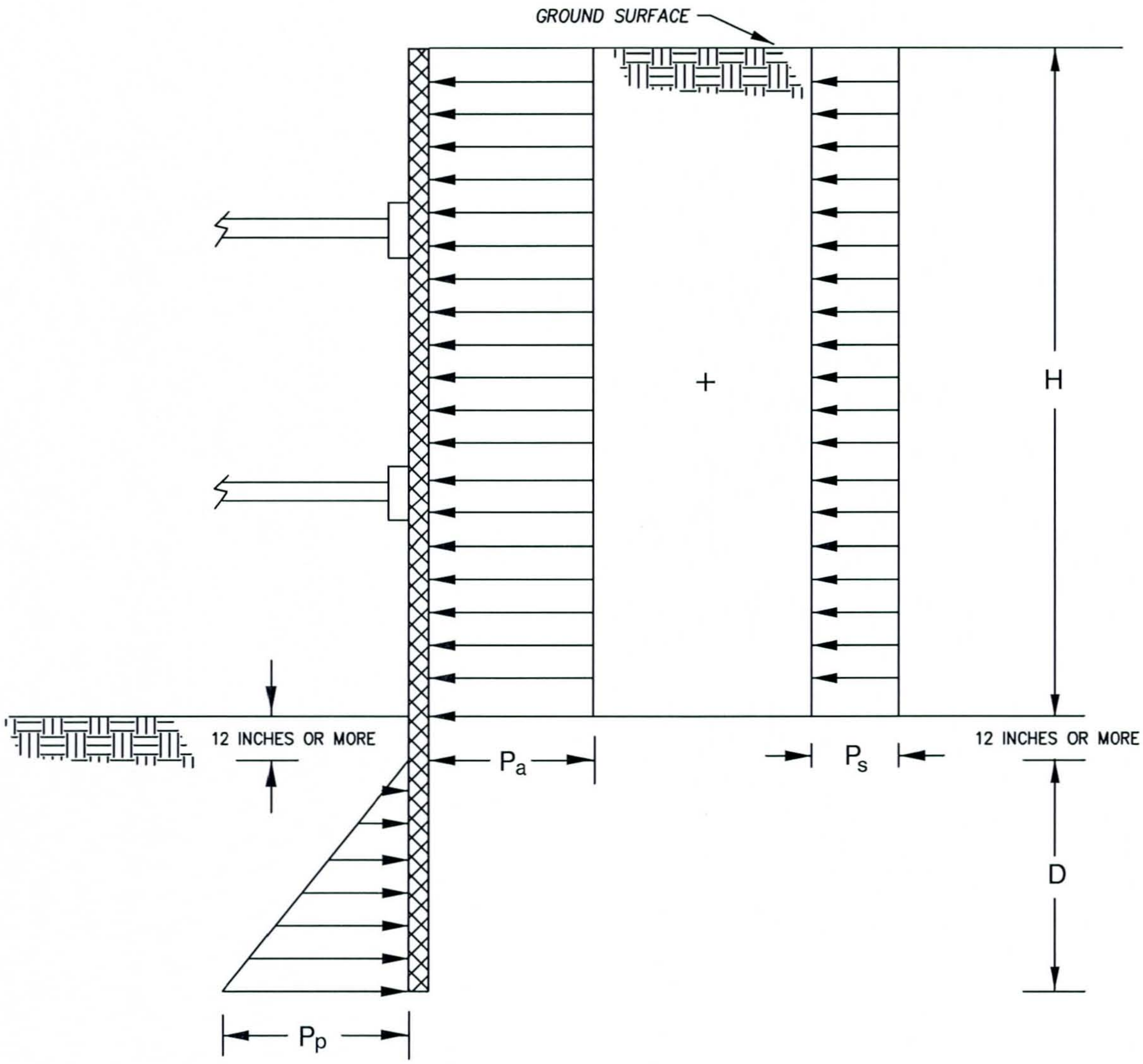
NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   $P_a = 35\ H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   $P_s = 120$ psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   $P_p = 300\ D$ psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. H AND D ARE IN FEET

NOT TO SCALE

1850dtl1007a

Ninyo & Moore		LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING	FIGURE
PROJECT NO.	DATE	BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	3
601850001	9/08		

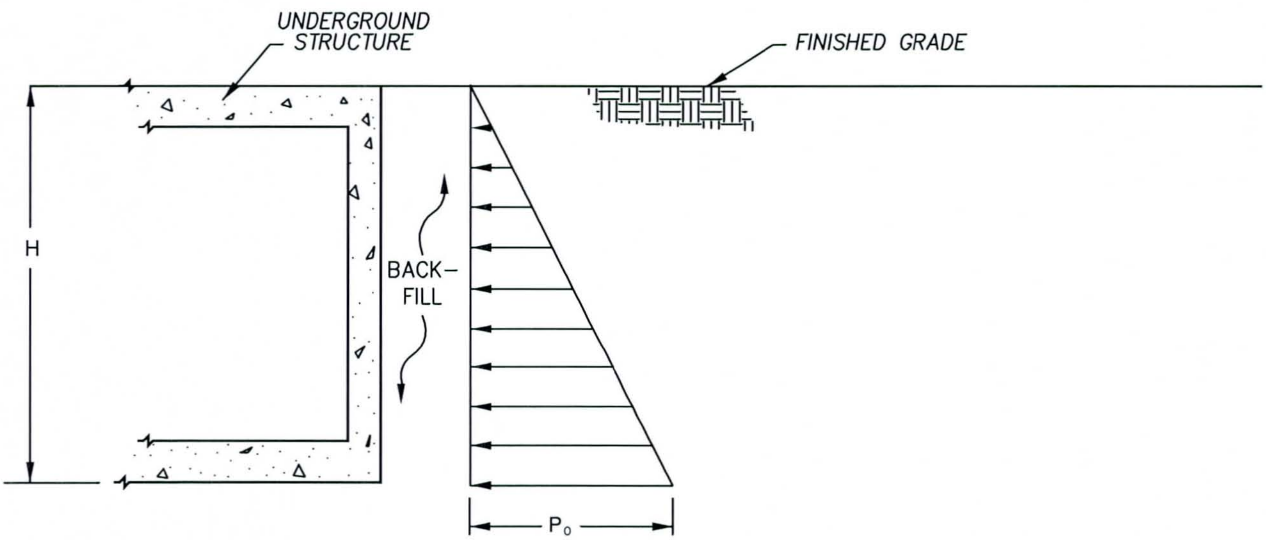


NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   $P_a = 22\,H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   $P_s = 120$ psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   $P_p = 345\,D$ psf
4. ASSUMES GROUNDWATER NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

Ninyo & Moore		LATERAL EARTH PRESSURES FOR BRACED EXCAVATION (GRANULAR SOIL)	FIGURE
PROJECT NO.	DATE	BETHANY HOME ROAD OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	4
601850001	9/08		



STATIC PRESSURE

NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_0
   $P_0 = 55\ H$ psf

2. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED

3. H IS IN FEET

4. ASSUMES NO GROUNDWATER

1850dtl1007b

NOT TO SCALE

Ninyo & Moore		LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES	FIGURE
PROJECT NO.	DATE	BETHANY HOME ROAD OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	5
601850001	9/08		

Ninyo & Moore

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test Spoon

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test spoon sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The spoon was driven up to 18 inches into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-99. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the spoon, bagged, sealed, and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-99. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

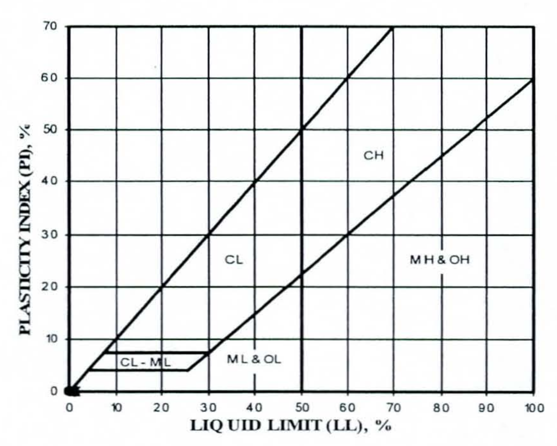
U.S.C.S. METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (More than 1/2 of soil >No. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction <No. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 1/2 of soil <No. 200 sieve size)	SILTS & CLAYS Liquid Limit <50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit >50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

GRAIN SIZE CHART

CLASSIFICATION	RANGE OF GRAIN SIZE	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL **Coarse** **Fine**	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND **Coarse** **Medium** **Fine**	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

PLASTICITY CHART



Ninyo & Moore | U.S.C.S. METHOD OF SOIL CLASSIFICATION

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	BORING LOG EXPLANATION SHEET
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	ALLUVIUM: Solid line denotes unit change.
								Dashed line denotes material change.
								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Sheared Bedding Surface
15								
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG		
EXPLANATION OF BORING LOG SYMBOLS		
PROJECT NO.	DATE Rev. 01/03	FIGURE

DATE DRILLED	08/14/07	BORING NO.	B-1
GROUND ELEVATION	--	SHEET	1 OF 2
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.)		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	KJT		

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 11 inches thick.
						GP-GM	AGGREGATE BASE: Approximately 7 inches thick. Brown, damp, medium dense, fine to coarse GRAVEL with silt and sand.
						GP	FILL: Brown, damp, medium dense, poorly graded GRAVEL with sand.
	Driven	28					
						SP	Brown, damp, dense, poorly graded SAND; few fine to coarse gravel.
	Driven	22					
5	Bulk						
	Driven	22				SW-SM	ALLUVIUM: Brown, damp, medium dense, well-graded SAND with silt and gravel.
	Driven	50/5"					Very dense.
10							
	Driven	14	2.6	114.6			Loose to medium dense.
15							
						SM	Brown, damp, loose, silty fine to medium SAND; trace fine gravel.
	Driven	5					
20							

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BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-1

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 08/14/07 BORING NO. B-1 GROUND ELEVATION -- SHEET 2 OF 2 METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.) DRIVE WEIGHT 140 lbs. (Automatic) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY KJT **DESCRIPTION/INTERPRETATION**
20								Total Depth = 20 feet. Groundwater not encountered during drilling. Backfilled and asphalt patched on 08/14/07 promptly after completion of drilling. Groundwater, though not encountered at the time of drilling, may rise to higher levels due to seasonal variations in precipitation and several other factors as discussed in the report.
25								
30								
35								
40								

Ninyo & Moore

BORING LOG		
BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA		
PROJECT NO.	DATE	FIGURE
601850001	9/08	A-2

DATE DRILLED 08/14/07 BORING NO. B-2

GROUND ELEVATION -- SHEET 1 OF 3

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY KJT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ML	ALLUVIUM: Brown, damp, medium dense, sandy SILT.
			8					
			11	20.5	100.0			Moist, loose.
5								
			5					
			11	7.9	105.7			Damp.
10								
			6					
15								
			31					Dense; scattered caliche nodules.
20								

Ninyo & Moore

BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-3

DATE DRILLED 08/14/07 BORING NO. B-2

GROUND ELEVATION -- SHEET 2 OF 3

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY KJT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							ML	ALLUVIUM: (continued) Brown, damp, dense, fine to medium sandy SILT; scattered caliche nodules.
							SC	Brown, damp, dense, clayey fine to medium SAND.
25			32					
			54				SP	Brown, damp, dense, poorly graded SAND; trace fine to coarse gravel.
30								
35			43					Very dense; few fine to coarse gravel.
							SC	Brown, damp, very dense, clayey fine to coarse SAND; trace fine gravel; scattered caliche nodules.
40			74/11"					

Ninyo & Moore

BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-4

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION

DATE DRILLED 08/14/07 BORING NO. B-2

GROUND ELEVATION -- SHEET 3 OF 3

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY KJT

DEPTH (feet)	DESCRIPTION/INTERPRETATION
40	Total Depth = 39.9 feet. Groundwater not encountered during drilling. Backfilled on 08/14/07 promptly after completion of drilling. Groundwater, though not encountered at the time of drilling, may rise to higher levels due to seasonal variations in precipitation and several other factors as discussed in the report.
45	
50	
55	
60	

Ninyo & Moore

BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-5

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	FILL: Brown, damp, loose, clayey SAND.
			19					
							CL	ALLUVIUM: Brown, damp, stiff, sandy CLAY.
			9					
5								
			6	18.8	89.1			Moist; firm to stiff.
							SM	Brown, damp, loose, silty fine SAND.
			4					
10								
							CL	Brown, moist, firm to stiff, CLAY.
			6					
15								
			1/18"					Very soft.
20								

DATE DRILLED 08/20/07 BORING NO. B-3

GROUND ELEVATION -- SHEET 1 OF 3

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY JLS LOGGED BY JLS REVIEWED BY KJT

Ninyo & Moore

BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-6

DATE DRILLED	08/20/07	BORING NO.	B-3
GROUND ELEVATION	--	SHEET	2 OF 3
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.)		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	JLS	LOGGED BY JLS	REVIEWED BY KJT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							CL	ALLUVIUM: (continued) Brown, moist, very soft, CLAY.
		■	38	19.6	106.0			Hard; scattered caliche nodules.
25								
							SC	Brown, moist, medium dense, clayey SAND; trace fine gravel.
		▼	15					
30								
							GP	Brown, damp, dense, poorly graded GRAVEL with sand; cobbles and possible boulders.
		■	50					
35								
		▼	50/5"					Very dense.
								Total Depth = 39.4 feet.
40								

Ninyo & Moore

BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-7

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 08/20/07 BORING NO. B-3 GROUND ELEVATION -- SHEET 3 OF 3 METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.) DRIVE WEIGHT 140 lbs. (Automatic) DROP 30" SAMPLED BY JLS LOGGED BY JLS REVIEWED BY KJT DESCRIPTION/INTERPRETATION
40								Groundwater not encountered during drilling. Backfilled on 08/20/07 promptly after completion of drilling. Groundwater, though not encountered at the time of drilling, may rise to higher levels due to seasonal variations in precipitation and several other factors as discussed in the report.
45								
50								
55								
60								

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BORING LOG		
BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA		
PROJECT NO.	DATE	FIGURE
601850001	9/08	A-8

DATE DRILLED	08/20/07	BORING NO.	B-4
GROUND ELEVATION	--	SHEET	1 OF 2
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.)		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	KJT		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 2 inches thick.
							GP-GM	AGGREGATE BASE: Approximately 3.5 inches thick. Brown, damp, medium dense, fine to coarse GRAVEL with silt and sand.
							CL	ALLUVIUM: Brown, damp, stiff, CLAY with sand.
			7					
			19					Very stiff.
5								
							ML	Brown, damp, loose, sandy SILT.
			5					
			21	10.5	102.1		SM	Brown, damp, medium dense, silty fine to medium SAND.
10								
							ML	Brown, damp, loose, sandy SILT.
			6					
15								
							CL	Brown, damp, stiff, sandy CLAY.
			7					
20								

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BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-9

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 08/20/07 BORING NO. B-4 GROUND ELEVATION -- SHEET 2 OF 2 METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.) DRIVE WEIGHT 140 lbs. (Automatic) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY KJT DESCRIPTION/INTERPRETATION
20								Total Depth = 20 feet. Groundwater not encountered during drilling. Backfilled on 08/20/07 promptly after completion of drilling. Groundwater, though not encountered at the time of drilling, may rise to higher levels due to seasonal variations in precipitation and several other factors as discussed in the report.
25								
30								
35								
40								

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BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-10

DATE DRILLED	08/20/07	BORING NO.	B-5
GROUND ELEVATION	--	SHEET	1 OF 2
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.)		
DRIVE WEIGHT	140 lbs. (Automatic)	DROP	30"
SAMPLED BY	DM	LOGGED BY	DM
REVIEWED BY	KJT		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4 inches thick.
							GP	AGGREGATE BASE: Approximately 8 inches thick.
			20				CL	FILL: Brown, damp, very stiff, sandy CLAY.
			5				CL	ALLUVIUM: Brown, damp, firm to stiff, sandy CLAY.
5								
			3	14.4	96.9		ML	Brown, moist, very loose, sandy SILT.
			3				CL	Brown, damp, soft to firm, sandy CLAY.
10								
			11				SM	Brown, damp, loose, silty SAND with gravel.
15								
			5				ML	Brown, damp, loose, sandy SILT.
20								

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BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-11

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20								Total Depth = 20 feet. Groundwater not encountered during drilling. Backfilled on 08/20/07 promptly after completion of drilling. Groundwater, though not encountered at the time of drilling, may rise to higher levels due to seasonal variations in precipitation and several other factors as discussed in the report.
25								
30								
35								
40								

DATE DRILLED 08/20/07 BORING NO. B-5

GROUND ELEVATION -- SHEET 2 OF 2

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (Enviro-Drill Inc.)

DRIVE WEIGHT 140 lbs. (Automatic) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY KJT

Ninyo & Moore

BORING LOG

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA

PROJECT NO.	DATE	FIGURE
601850001	9/08	A-12

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APPENDIX B

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-00. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937-04. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis

Gradation analyses were performed on selected representative soil samples in general accordance with ASTM D 422-63 (02). The grain-size distribution curves are presented on Figures B-1 through B-5. The test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System.

Atterberg Limits

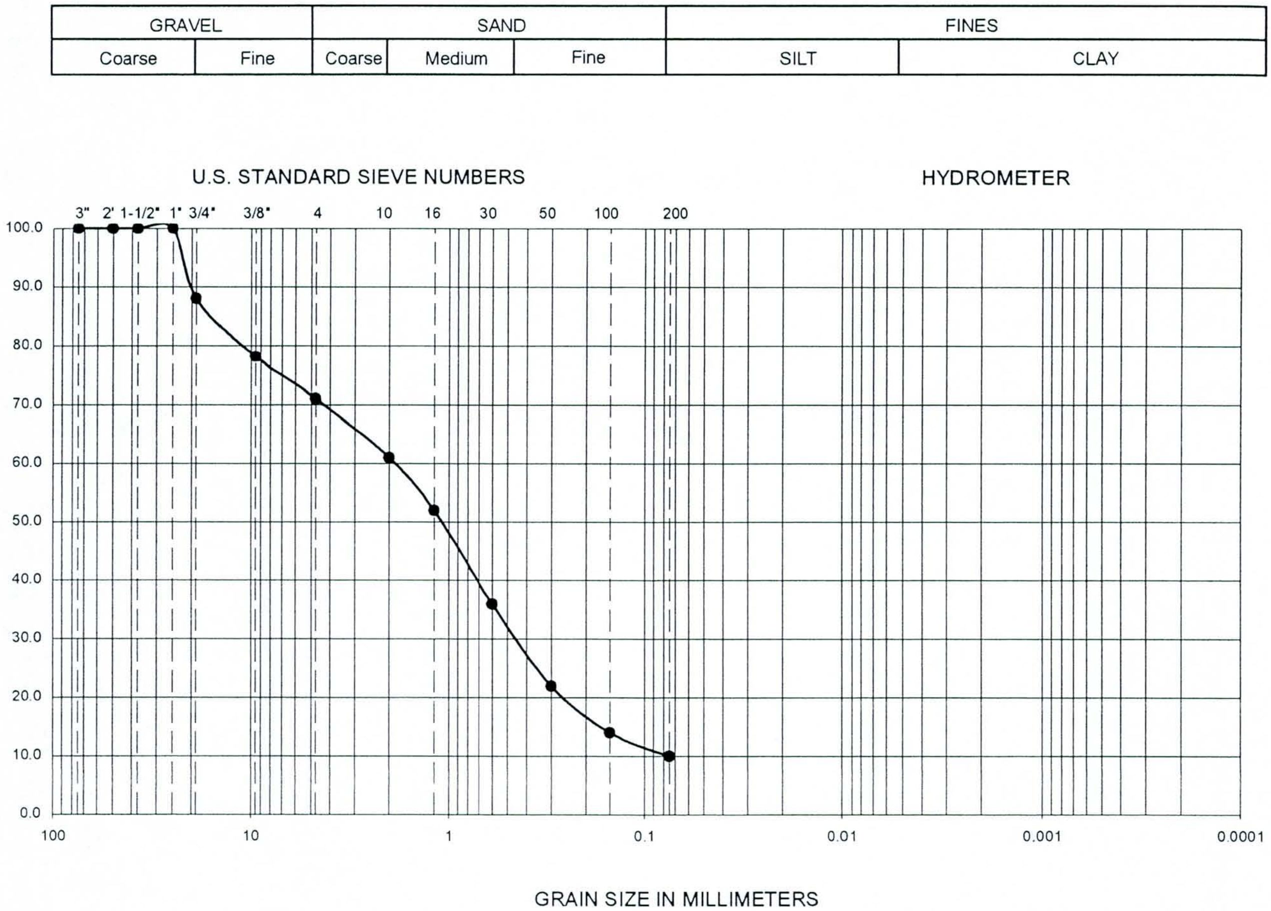
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318-05. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results are incorporated in the logs in Appendix A and are shown on Figure B-6 in Appendix B.

Maximum Dry Density and Optimum Moisture Content Tests

The maximum dry density and optimum moisture content of selected representative soil samples were evaluated in general accordance with ASTM D 698-00. The results of these tests are summarized on Figure B-7.

Soil Corrosivity Tests

Soil pH and minimum resistivity tests were performed on representative samples in general accordance with Arizona Test 236b. The chloride content of selected samples was evaluated in general accordance with Arizona Test 736. The sulfate content of selected samples was evaluated in general accordance with Arizona Test 733. The test results are presented on Figure B-8.

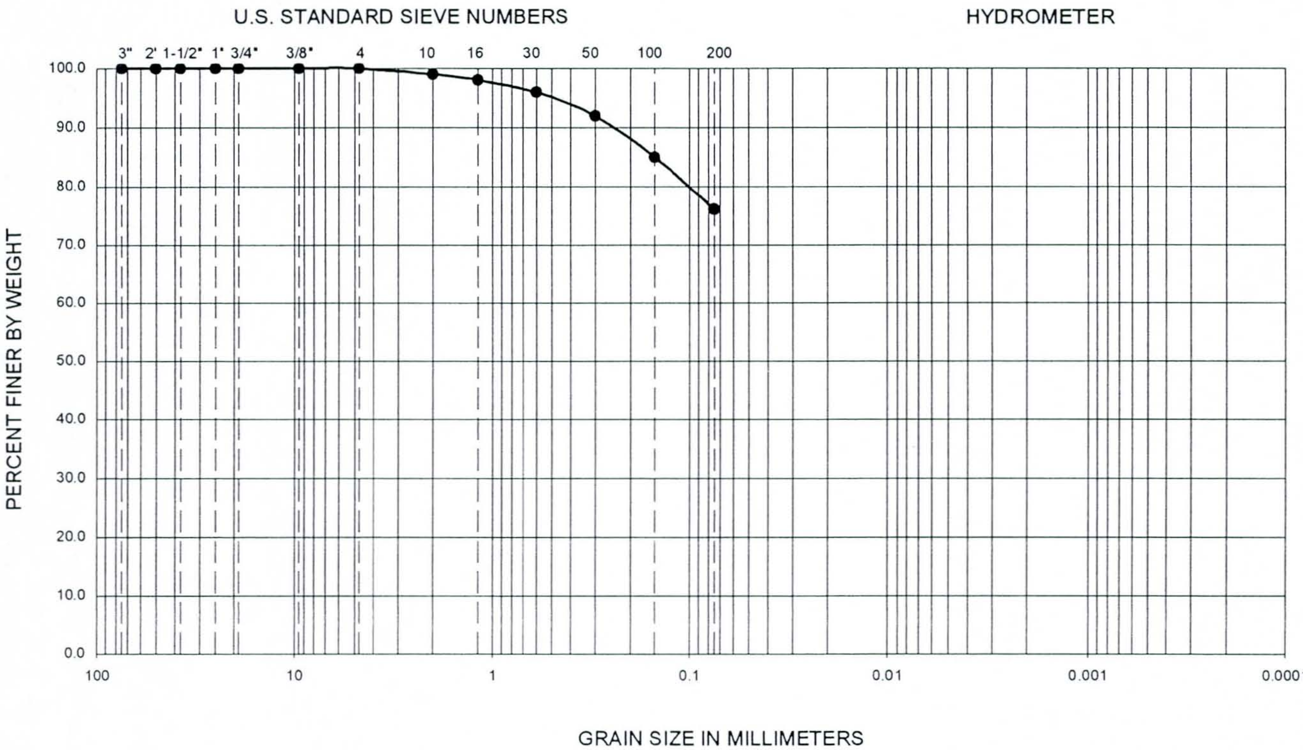


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-1	6-7.5	--	--	--	0.08	0.46	1.90	25.3	1.5	10	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	B-1
601850001	9/08		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

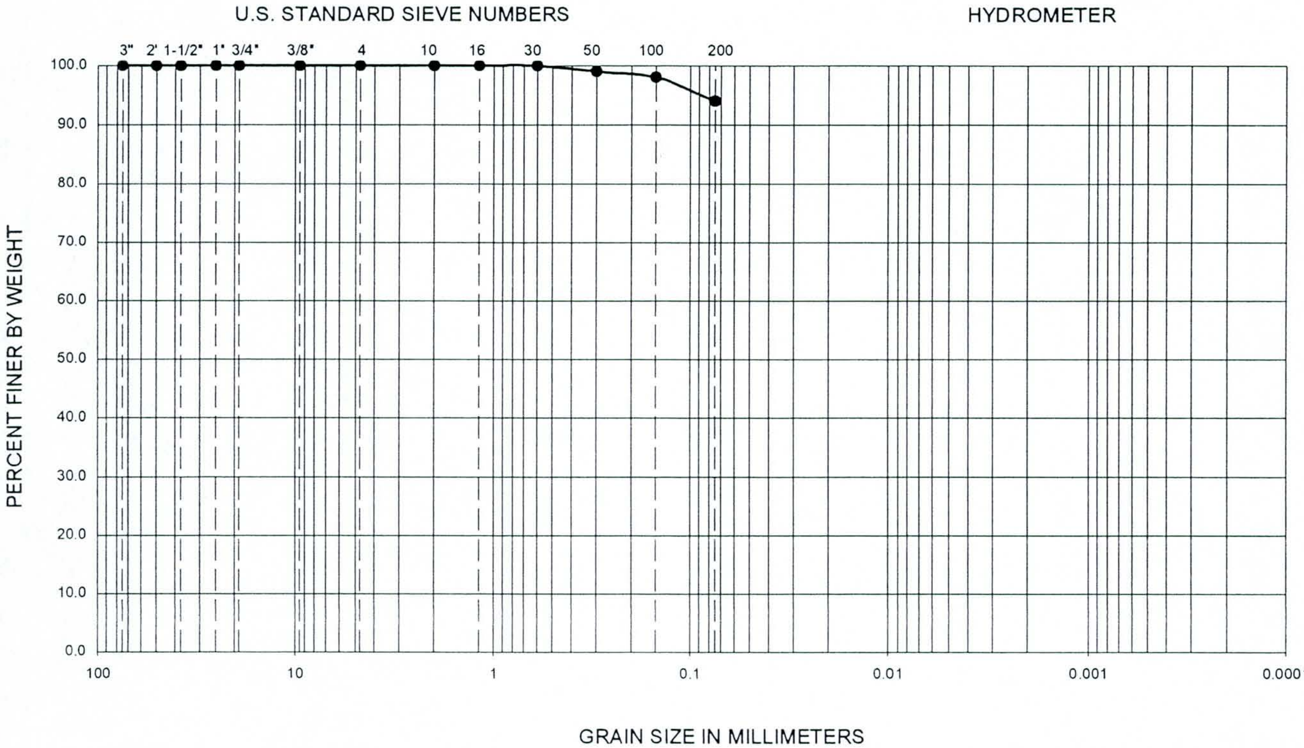


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-2	6-7.5	--	--	--	--	--	--	--	--	76	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	B-2
601850001	9/08		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

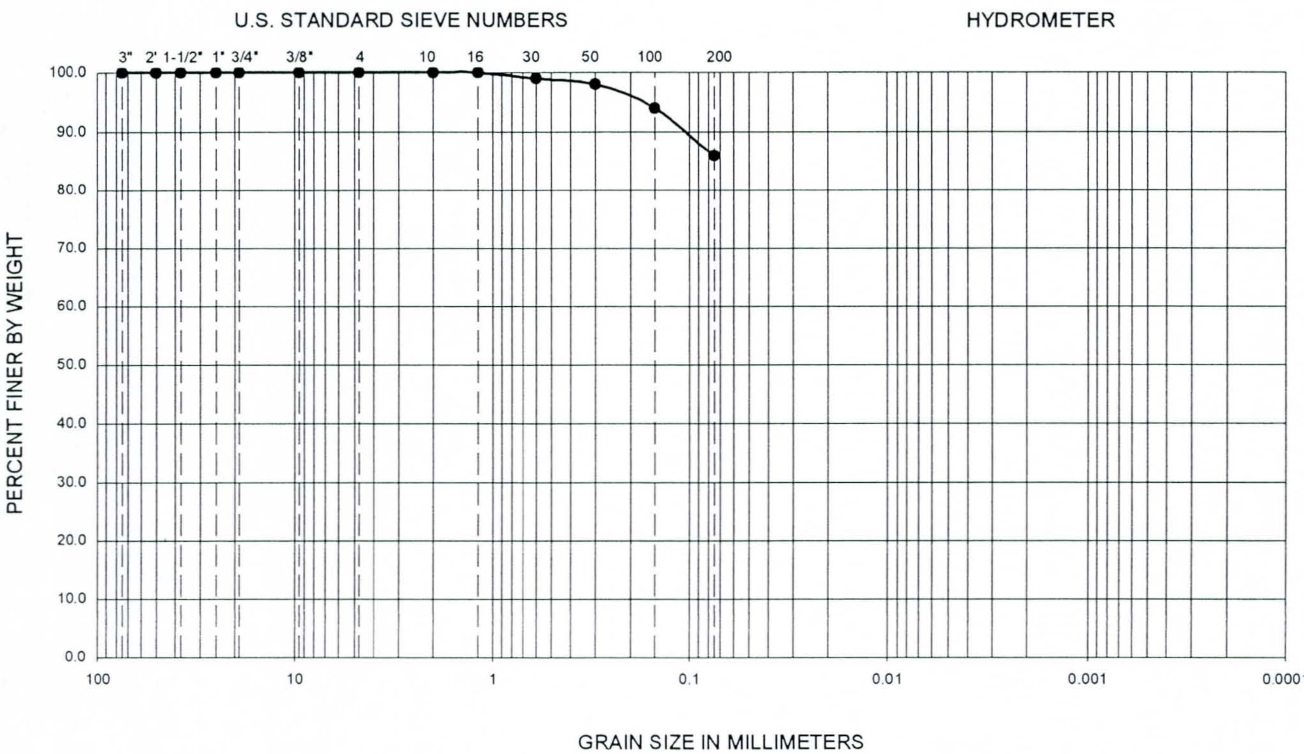


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-3	3.5-5	40	18	22	--	--	--	--	--	94	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	B-3
601850001	9/08		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

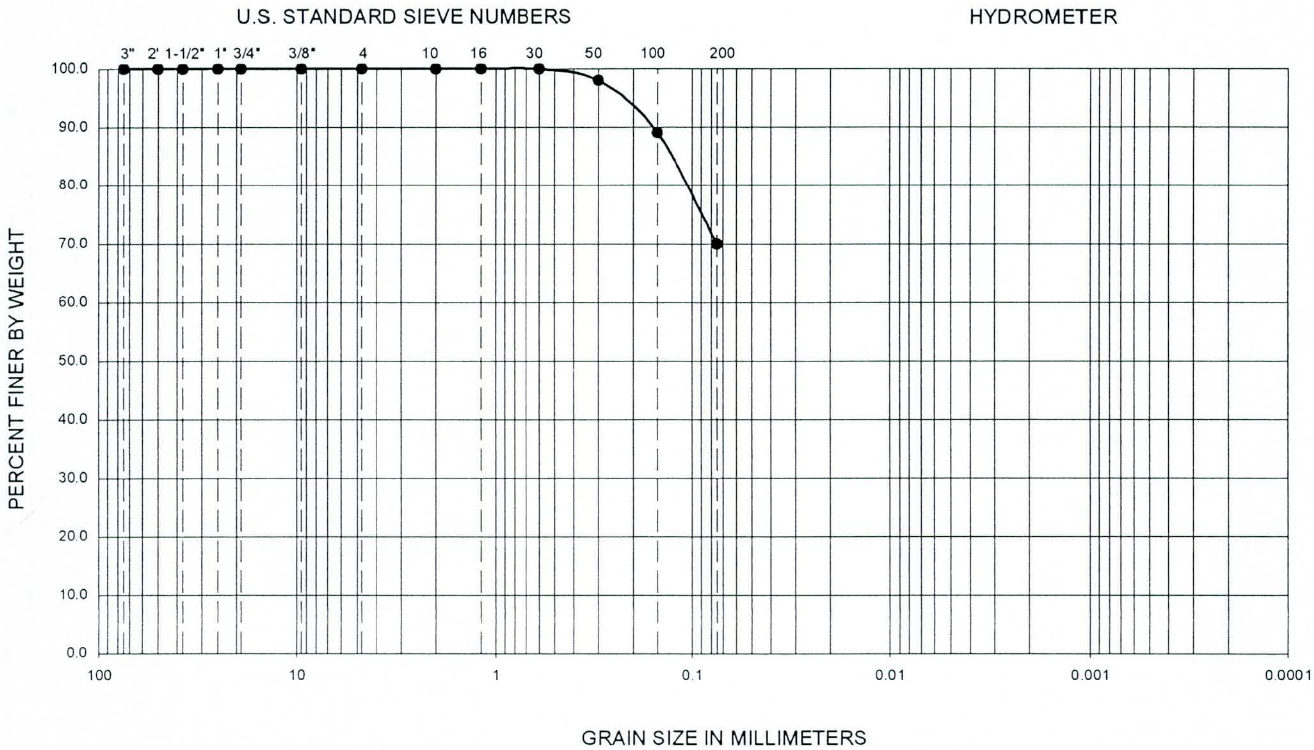


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-4	3.5-5	44	18	26	--	--	--	--	--	86	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

Ninyo & Moore	GRADATION TEST RESULTS		FIGURE
PROJECT NO.	DATE	BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	B-4
601850001	9/08		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

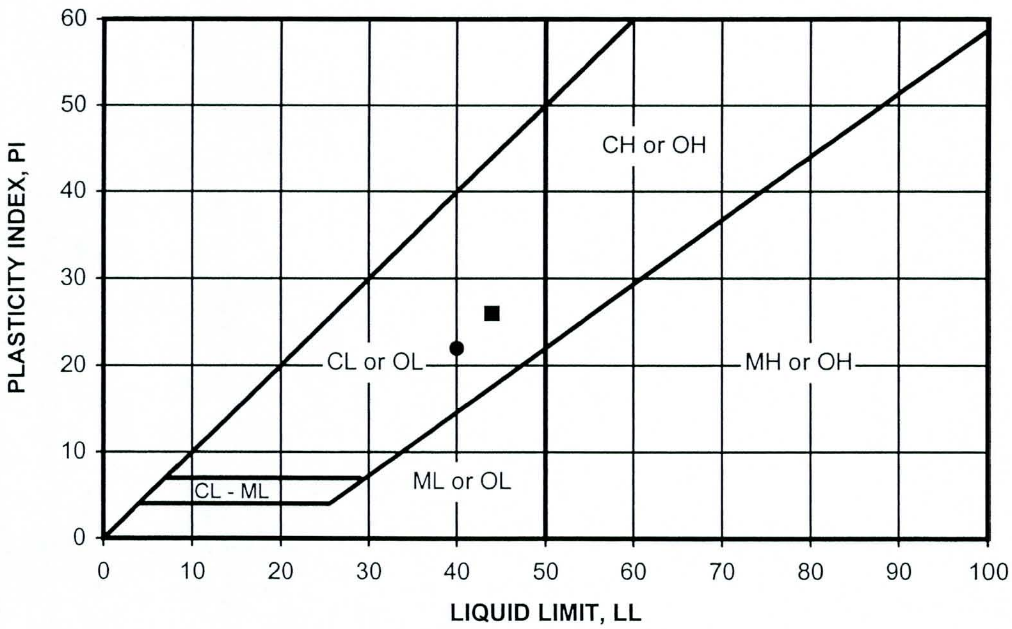


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-5	6-7.5	--	--	--	--	--	--	--	--	70	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

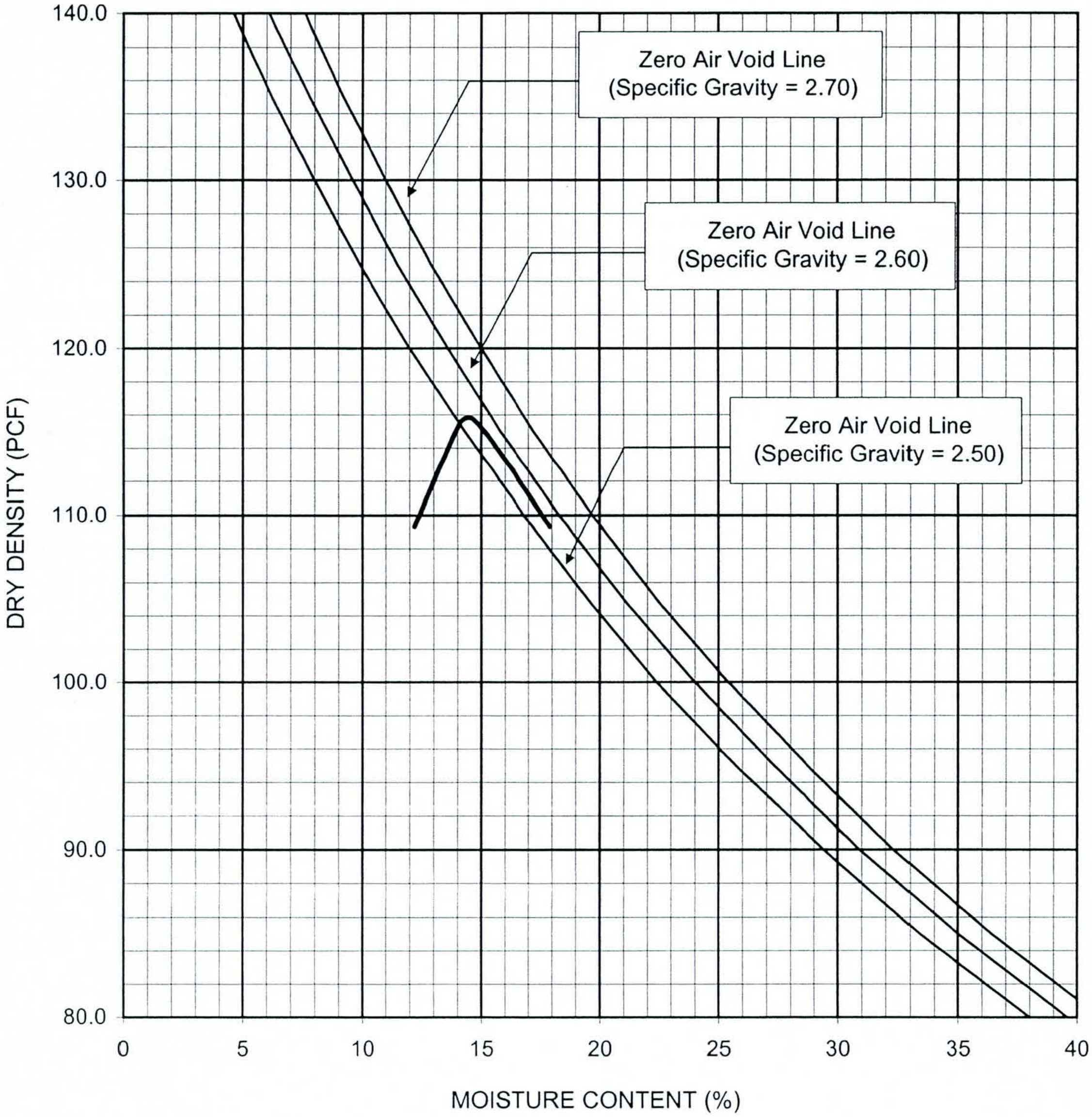
Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	**B-5**
601850001	9/08		

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-3	3.5-5	40	18	22	CL	CL
■	B-4	3.5-5	44	18	26	CL	CL



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-05

Ninyo & Moore		ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO.	DATE	BETHANY HOME OUTFALL CHANNEL REACH D PHOENIX, ARIZONA	B-6
601850001	9/08		



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-3	0-3	SC	115.8	14.6
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718-87)			N/A	N/A

PERFORMED IN GENERAL ACCORDANCE WITH ☐ ASTM D 1557-02 ☑ ASTM D 698-00a METHOD ☑ A ☐ B ☐ C

Ninyo & Moore	PROCTOR DENSITY TEST RESULTS	FIGURE
PROJECT NO. / DATE	BETHANY HOME OUTFALL CHANNEL REACH D	B-7
60185001 / 9/08	PHOENIX, ARIZONA	

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	5-10	8.5	2,394	8	0.001	43
B-3	0-3	7.8	752	57	0.006	146

[1] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 236b
[2] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 733
[3] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 736

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CORROSIVITY TEST RESULTS

FIGURE

B-8

PROJECT NO.	DATE
601850001	9/08

BETHANY HOME OUTFALL CHANNEL REACH D
PHOENIX, ARIZONA